U.S. DEPARTMENT OF COMMERCE
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PB-258 819

Upgrading Existing Wastewater Treatment Plants. Case Histories of Biological Process Modifications

Hazen and Sawyer, New York

Prepared for

Environmental Protection Agency, Cincinnati, Ohio Office of Technology Transfer

Nov 72

295021

PB 258 819

UPGRADING EXISTING WASTEWATER TREATMENT FACILITIES

CASE HISTORIES OF BIOLOGICAL PROCESS MODIFICATIONS

PREPARED FOR
ENVIRONMENTAL PROTECTION AGENCY
TECHNOLOGY TRANSFER DESIGN SEMINAR

CHICAGO, ILLINOIS
NOVEMBER 28 - 30, 1972

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

HAZEN AND SAWYER
ENGINEERS

NEW YORK, N.Y. HOLLYWOOD, FLA.

57

BIBLIOGRAPHIC DATA SHEET	1. Report No.	2.	3. Recipient's Accession No. PB-258 819
4. Title and Subtitle Upgrading Existing Wastewater Treatment Plants. Case Histories of Biological Process Modifications.			5. Report Date Nov. 72 6.
7. Author(s)			8. Performing Organization Rept. No.
9. Performing Organization Name and Address Hazen and Sawyer, New York.			10. Project/Task/Work Unit No. 11. Contract/Grant No.
12. Sponsoring Organization Name and Address Environmental Protection Agency, Cincinnati, Ohio. Office of Technology Transfer			13. Type of Report & Period Covered 14.

15. Supplementary Notes
Presented at the Environmental Protection Agency Technology Transfer Program Design Seminar Held at Chicago, Illinois on November 28-30, 1972.

16. Abstracts

The case histories described in this report relate to upgrading the performance of activated sludge and trickling filter plants through modification to the biological processes. In these processes the biochemical reaction rates depend on the concentration of reactants (wastes) and of the appropriate microorganisms capable of feeding on the wastes.

17. Key Words and Document Analysis. 17a. Descriptors

*Sewage treatment,
*Activated sludge process,
*Trickling filters,

Upgrading,
Microorganism control(sewage treatment)
Clarification,
Aeration,
Performance,
Specifications,

North Carolina,
New York,
California.

17b. Identifiers/Open-Ended Terms

*Sewage treatment plants.

17c. COSATI Field/Group 13B

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
National Technical Information Service Springfield, Va. 22161	UNCLASSIFIED	57p
	20. Security Class (This Page) UNCLASSIFIED	22. Price 4.50/3.00

FORM NTIS-35 (REV. 10-73) ENDORSED BY ANSI AND UNESCO. THIS FORM MAY BE REPRODUCED USCOMM-DC 8265-P74

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WASTEWATER TREATMENT PLANTS
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Chicago, Ill., November 28-30, 1972

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PREPARED FOR DESIGN SEMINAR

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INTRODUCTION

described in this report

The case histories herein relate to upgrading the performance of activated sludge and trickling filter plants through modification to the biological processes.

In these processes the biochemical reaction rates depend on the concentration of reactants (wastes) and of the appropriate microorganisms capable of feeding on the wastes.

For high degrees of treatment, rates of BOD removal per unit weight of microorganism drop off. Equilibrium is reached when the rate of BOD removal is balanced by BOD from lysis products of dead microorganisms. Equilibrium soluble BOD levels vary from system to system but typically are above 5 mg/l. For biological systems which include effluent filtration or other effective means of removing fine solids from the effluent, the minimum equilibrium soluble BOD level provides the target for optimization.

In actual systems the following factors may limit performance to less than optimum.

1. Inadequate hydraulic retention or contact time

This is generally a significant limiting factor only where:

a. High removals are required. (Effluent BOD $<$ 20 ppm).

b. System operates on short contact times e.g. trickling filter or contact stabilization (also for a fraction of flow in completely mixed or step aeration systems).

c. Wastes contain constituents not readily adsorbed on microbial flocs or slimes.

Most BOD constitutents of domestic sewage are adsorbed rapidly provided the microorganisms in system have had adequate opportunity to assimilate previously adsorbed material.

Some BOD constitutents — particularly industrial wastes with high dissolved fractions— do not show rapid initial removal. Whether this is an inherent waste characteristic or is related to the population characteristics of the microorganisms has generally not been established for particular systems.

2. Solids Retention Time

This parameter determines the characteristics of the microbial population in a treatment system: types

of organisms present, reaction rates for particular waste constituents, settling characteristics, etc.

Solids retention time reflects the biological loading on the system (on an overall basis:lb BOD/day/lb MLVSS) the cell yield for the particular waste constituents and biological solids in the system, (on an overall basis: lb solids/lb BOD removed) and the rate of autooxidation of the biological solids. In general, systems with high loadings show low solids retention times.

Increasing solids retention time will in general, improve performance. In lightly loaded systems, however, increase in solids retention may actually reduce performance by:

(1) increasing opportunities for return of BOD to system from lysis of biological solids.

(2) reducing settleability of biological solids (especially in activated sludge systems.)

Hence to optimize performance, an operator needs not only treatment facilities of adequate capacity but

also provisions for control of the solids in the system (variable return sludge and waste sludge rates in activated sludge; variable recirculation in trickling filters,etc.).

Most systems in need of upgrading, however, suffer from inadequate solids retention times. This is especially true where a particular waste component is removed by only by relatively slow growing microorganisms. If the solids retention time in the system is less than the generation time of the particular microorganisms they will be washed or crowded out of the system. In this case the particular waste component will not be removed or oxidized at all. The most important example of this is the oxidation of ammonia in activated sludge or trickling filter systems. Typically no nitrification results in single stage systems unless loadings are low enough to permit long solids retention times.

Solids retention times may be increased by:

(1) increasing reactor volume

(2) increasing the concentration of biological material in the reactor.

Generally the only limit on increasing reactor volume is cost. Increasing reactor solids levels may involve no direct costs but may be limited, however, by the other related factors:

1) In activated sludge systems these include:
   - higher solids loadings imposed on final clarifiers.
   - ability to supply oxygen to the reactor and to the interior of sludge floc particles themselves.
2) In trickling filters or other fixed film reactors these include:
   - the maximum media surface per unit volume of reactor that can be employed without clogging (high area per unit volume requires fine media).
   - ventilation of reactor and ability to diffuse oxygen and waste constituents into biological films.

In increasing performance of a particular plant the foregoing factors must be examined and all those limiting

performance at the desired level must be modified. The following table indicates specific process modifications which should be considered to overcome particular limitations on system performance.

Modification or Addition	Where to Use	Remarks
Tighten process control	To minimize soluble BOD in aerator effluent and improve settling characteristics of sludge.	Requires flexibility in return, waste sludge and air rates, and requires control instrumentation.
Add final tank capacity	If solids loading on final tanks limits the solids level that can be maintained in aerator.	Check buildup of sludge in clarifiers at high flow rates.
Increase aerator solids level	To increase removals of waste constituents assimilated by slow growing microorganisms.	Consider effect on final tanks, and adequacy of oxygen sypply.
Increase air supply or improve its distribution	Where dissolved oxygen cannot be maintained above 1-3 ppm at all loads and locations.	Consider oxygen aeration.
Increase contact time	In contact stabilization system where tests indicate rapid initial removal is below level desired.	Check effects on settling characteristics of sludge.
Divide reactor into multiple compartments in series	Where short circuiting has significant effect on BOD levels in aerator effluent.	

Modification or Addition	Where to Use	Remarks
Contact stabilization	For upgrading overloaded plug flow or multi-compartmented tanks where initial removal is adequate.	Avoid in small plants with wide diurnal flow variations.
Step aeration	For upgrading overloaded plug flow or multi-compartmented tanks to higher removals than possible with contact stabilization.	
Completely mixed activated sludge	For upgrading any conventional activated sludge system and for conversion of small contact stabilization plants.	Piping changes will be very costly for long plug flow tanks; step aeration preferable in such cases.

Trickling Filters

Modification of Addition	Where to Use	Remarks
Positive Ventilation	For strong wastes and deep filters where natural draft is not adequate to provide oxygen needed.	Check with media manufacturers.
Media Change	In rock filters where increase in media surface (and biomass) will provide needed performance improvement.	Synthetic media are available with from 2-4 times the surface-to-volume ratio of stone. Fine media cannot be used for high loadings due to clogging.
Increase Recirculation	For lightly loaded filters where initial removal capacity is not fully utilized in one-pass contact time.	May also be needed for dilution to maintain aerobic conditions in heavily loaded filters.
Activation (Sludge recirculation)	Where more biomass is needed to increase treatment.	Requires thorough pilot study, (Refer to Water and Wastewater Engineering Aug. 1970 p. 34).

Unit Added	Location	Remarks
High rate tirck-ling filter (roughing)	Ahead of overloaded trickling filter or activated sludge units.	Intermediate settling not needed. Perform-ance quite temperature sensitive.
Activated sludge		May be designed for nitrification. Requires extra set of settling tanks.
	After trickling filter or activated sludge plant.	Advantageous when first unit can be converted to "high rate" (high loading) operation.
	Ahead of trickling filter.	Limit to special cases such as industrial pre-treatment.
	Ahead of activated sludge.	Use where existing facilities are suitable for final stage.
Nitrification trickling filter	After settling tanks of existing secondary system	Provides polishing and nitrification. Because of low sludge yield effluent probably can be applied directly to granular media filters for solids removal. Ad-vantageous where effluent filters are to be included in upgrading.

CASE HISTORY I

GREENSBORO, NORTH CAROLINA
SOUTH BUFFALO CREEK WASTEWATER TREATMENT PLANT

The original South Buffalo Creek Sewage Treatment Plant was built in 1931 for an ultimate population of 37,000, an average flow of 3.25 million gallons per day and a wet weather flow of 6.5 million gallons per day. The plant was generally adequate for the design load except that the final settling tank, sludge digestion tank and sludge drying beds were considerably smaller than would be called for by modern practice. These deficiencies resulted in somewhat less satisfactory treatment than would ordinarily be expected with this type of plant.

Facilities in the original plant consisted of screens, detritors, single primary and final settling tanks, two fixed-nozzle trickling filters, a digester and sludge drying beds.

By 1956 the flow to the plant had increased to an average of 4.0 million gallons per day, of which 1.5 million gallons per day was industrial waste. The total load to the plant was equivalent to that from 52,000 people, and the treatment efficiency was inadequate.

Among the industries contributing to the plant were textile dyeing and finishing plants, an abattoir, a meat packing plant, a chemical manufacturing plant, poultry processing plants, and metal plating plants.

The plant was upgraded in 1957-58 by replacement of screening and grit removal facilities, addition of one primary settling tank, two trickling filters, two final tanks, one digester, conversion of the existing final tank to chlorine contact, and replacement of the drying beds with a vacuum filter installation.

Between 1958 and 1964, the industrial BOD contribution to the plant increased much more rapidly than expected. The percent of BOD from industrial sources rose from 35% to over 65% and the overall plant BOD removal efficiency fell from 90% to 70%. To better handle the industrial loads one of the original fixed nozzle trickling filters was converted into two aeration basins in 1964-66.
Conversion involved removal of media and installation of platform mounted mechanical aerators. The trickling filters installed in 1957 were retained as roughing filters ahead of the activated sludge facilities.

Since the initial conversion, BOD removals have averaged above 90% and have never dropped below 80%.

In 1970, to meet increased loads, the remaining fixed-nozzle filter was converted to aeration basins using floating aerators.

The current upgrading project includes preaeration; chemical addition for phosphorus removal; special odor control measures; improved sludge handling ; effluent polishing with deep bed filters . BOD and suspended solids removals in excess of 98% are expected.

Design parameters of the various plant additions are detailed on Table I-1; typical plant operating data are given on Table I-2; capital costs of plant upgrading are outlined on Table I-3. Flow diagrams for various stages in development of the plant are shown in Figures I-1 to I-5.

Included is a possible future upgrading to provide nitrogen removal, should this be required. This might involve conversion of the 1957 trickling filters to provide nitrification after the activated sludge treatment and addition of biological denitrification.

TABLE I-1

UPGRADING OF SOUTH BUFFALO STP

DESIGN PARAMETERS

Item	1931 (Original Plant Design)	1957	1964-66	1970	1972 Proposals
A. Design Parameters					
Design Av. Flow (MGD)	3.25	8.0	8.0	8.0	10.7
Design Peak Flow (MGD)	6.50	16.0	16.0	16.0	21.4
Design BOD (lb/day)		21,700			35,500
Design SS (lb/day)		18,800			27,000
Phosphorus (lb/day)	-	-	-	-	900
B. Wastewater Treatment					
1. Screens	Mechanical Bar Screen	Existing Bar Screen Replaced by Back-raked Mechanical Bar Screen with Hand-raked Bypass Screen	No Change	No Change	Replace Mechanical Bar Screen by New Heavy-Duty Unit
2. Grit Chambers	Two Detritors	Detritors Replaced by Aerated Grit Chamber. Available Air Supply: 50 CFM Grit Removal: Tubular Conveyor Discharging to Grit Box	No Change	No Change	Replace Grit Removal Mechanism with New Heavy-Duty Bucket-type Grit Collecting and Elevating Mechanism
3. Pre-Aeration Tank	-	-	-	-	Volume: 1.1 MG Available Air Supply: 4,000 CFM
4. Primary Settling Tanks	Square Tank. Dimensions: 70'x70'x10' SWD Surface Area: 4,900 sq.ft. Overflow Rate: 670 gpd/sq.ft.	Old Square Tank Retained. New Round Tank (80' dia.x9' SWD) Added; Flow Equally Divided Between Tanks. Total Surface Area: 9,900 sq.ft. Overflow Rate: 800 gpd/sq.ft.	No Change	No Change	Enclose Primary Tanks with Positively Ventilated Building to Exhaust Air Through Odor-Control Units
5. Phosphorus Removal	-	-	-	-	Addition of Facilities to Add Lime, Alum or Iron Salts (Depending on Results of Pilot Plant Studies) Ahead of Primary Settling Tanks
6. Secondary Treatment	Two Square (200'x192'x7' Deep) Fixed-Nozzle, Stone Trickling Filters Media Volume: 540,000 cu.ft. Loading:	Square Filters Retained, Accepting Flow from Old Ordinary Tank-Loading 15 lb BOD/1,000 cu.ft. of Media/Day. Two New Round Filters (155'Dia.x7' Deep) Added to Accept Flow from New Primary Tank - Media Volume 260,000 cu.ft. loading 36 lb BOD/1,000 cu.ft. of Media per Day. New Filters have Recirculation Capacity of 9 MGD	Use of Square Filters Discontinued. Round Filters Operated as Roughing Filters, One Square Filter Converted to Two Aeration Basins with Combined Volume of 326,400 cu.ft. and Combined Aeration Capacity of 200 HP of Platform-Mounted Aerators.	Second Square Filter Converted to Provide Additional Two Aeration Basins with 360 HP Aeration Capacity, Giving Total Aeration Volume of 655,000 cu.ft. and Total Aeration Capacity of 560 HP.	No Change

TABLE I-1

UPGRADING OF SOUTH BUFFALO STP (continued)

DESIGN PARAMETERS

Item	1931 (Original Plant Design)	1957	1964-66	1970	1972 Proposals
7. Final Settling	One Rectangular Tank (50'x50'x10' SWD) Total Surface Area: 2,500 sq.ft. Overflow Rate: 1,300 gpd/ft^2	Old Final Settling Tank Converted to Chlorination Contact Tank (See Below). Two New Round (105' Dia. x9' SWD) Tanks Added to Handle Total Plant Flow Total Surface Area: 17,300 sq.ft. Overflow Rate: 460 gpd/sq.ft.	No Change	No Change	No Change
8. Chlorination	None	Contact Tank Volume:25,000 cu.ft. Chlorinator Capacity:4,000 lbs/day	No Change	No Change	No Change
9. Effluent Filtration	-	-	-	-	Preaeration Followed by Dual Media Filtration (Design Dependent on Results of Pilot Plant Studies)
C. Residue Treatment					
1. Sludge Thickening	-	-	-	-	Flotation Thickening of Activated Sludge Prior to Mixing with Primary Sludge Flotation Unit Surface Area: 250 sq.ft.
2. Sludge Digestion	Fixed-Cover Digester 60' dia. x 23' SWD Volume:	New Floating-Cover Digester (80' dia. x 30' SWD) Added as Primary Digester: Existing Fixed Cover Digester Retained as Secondary Unit	Digestion Discontinued. Digesters Retained as Sludge Holding Tanks	No Change	No Change
3. Sludge Dewatering	Sludge Drying Beds Area: 32,600 sq.ft.	New 11'-6" x 12' large Coil Spring Vacuum Filter (430 sq.ft. Surface Area) Added. Drying Beds Relegated to Standby Role	No Change	No Change	Recondition Existing Vacuum Filter and Accessories. Replace Filter Media. Provide Odor Control for Sludge Handling Building
4. Ultimate Disposal	Trucked to Landfill	Trucked to Landfill	Trucked to Landfill	Trucked to Incinerator	No Change

TABLE I-2

TYPICAL PERFORMANCE OF SOUTH BUFFALO CREEK SEWAGE TREATMENT PLANT

	1957-58		1964-66		1970		Proposed
	Before Upgrading	After Upgrading	Before Upgrading	After Upgrading	Before Upgrading	After Upgrading	After Upgrading
A. Characteristics of Raw Waste							
(i) Flow							
Average day (MGD)	3.7	4.5	4.9	6.6	6.8	8.9	10.7
Peak Hour (MGD)	10.0	11.5	11.5	16.0	16.0	18.0	21.0
(ii) BOD							
Average Day (mg/1)	380	310	290	300	390	390	400
Average Day (lb/day)	11,700	11,600	11,800	16,500	22,200	29,000	35,700
(iii) Suspended Solids							
Average Day (mg/1)	350	240	250	260	290	300	300
Average Day (lb/day)	10,800	9,100	10,200	14,100	16,500	22,300	26,800
B. Performance of Treatment Units							
(i) Average BOD Removal							
Primary (lb/day (%))	2,300 (20)	3,000 (25)	3,000 (25)	2,500 (15)	3,200 (15)	4,400 (15)	9,000 (25)
Trick. Filt. (lb/day (%))	4,700 (50)	7,400 (85)	5,200 (60)	5,800 (40)	6,600 (35)	7,600 (30)	8,000 (30)
Act. Sludge (lb/day (%))	--	--	--	6,500 (80)	9,000 (75)	14,100 (85)	16,000 (85)
Eff. Filt (lb/day (%))	--	--	--	--	--	--	2,000 (75)
Total (lb/day (%))	7,000 (60)	10,400 (90)	8,200 (70)	14,800 (90)	18,800 (85)	26,100 (90)	35,000 (98)
(ii) Average SS Removal							
Primary (lb/day (%))	4,300 (40)	4,500 (50)	5,100 (50)	4,200 (30)	5,000 (30)	7,800 (35)	13,400 (50)
Secondary (lb/day (%))	2,200 (35)	3,200 (70)	2,500 (50)	7,100 (70)	7,400 (65)	8,900 (60)	10,500 (80)
Eff. Filt. (lb/day (%))	--	--	--	--	--	--	2,300 (80)
Total (lb/day (%))	6,500 (60)	7,700 (85)	7,600 (75)	11,300 (80)	12,400 (75)	16,700 (75)	26,200 (98)
C. BOD Loading On Units (1000 lb/cu ft)							
Trickling Filter	17	15/36 (1)	15/36 (1)	54	73	95	103
Act. Sludge	--	--	--	25	38	26	29
D. Operating Costs ($/MG)	--	--	38.50	52.93	58.91	78.28	--

(1) Two-stage. See Table I-1.

TABLE I-3

SOUTH BUFFALO STP - CAPITAL COSTS OF UPGRADING

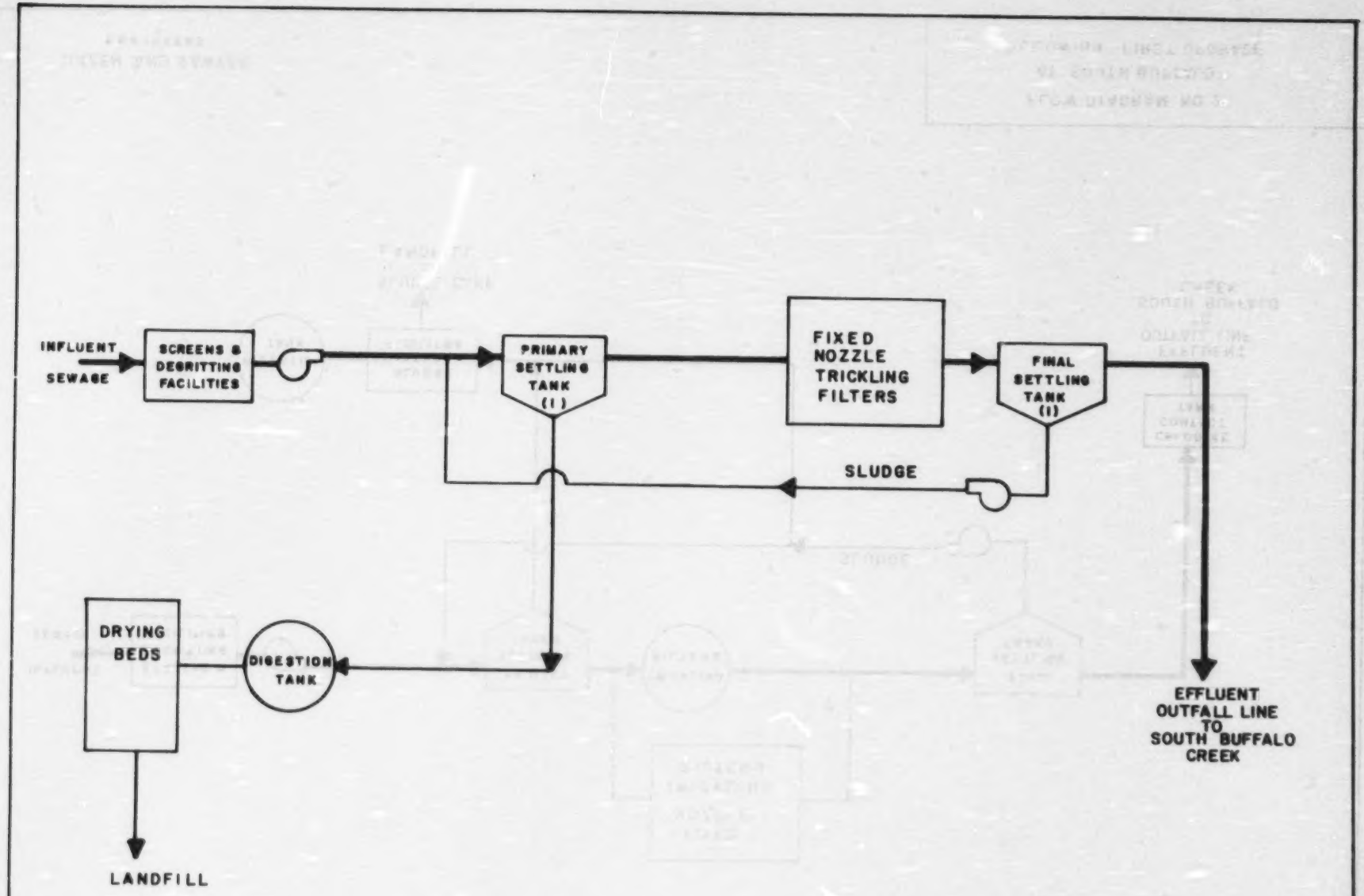
Date of Upgrading	Item	Capital Cost: $ Cost At Time Of Upgrading	Capital Cost: 1972 $ Cost
1957	Screening and degritting	45,000	110,000
	Rehabilitation of Pump & Control building	150,000	365,000
	Primary Settling	65,000	160,000
	Secondary Process	330,000	800,000
	Final Settling	135,000	330,000
	Chlorination Facilities	25,000	60,000
	Sludge Handling	300,000	730,000
	Miscellaneous	150,000	365,000
		1,200,000	2,920,000
1964-6	Modifications to Secondary Process	200,000	367,000
1970	Modifications to Secondary Process	100,000	133,000
1972 Proposal	Renovation of screens and degritting equipment		60,000
	Preaeration & Odor Control		160,000
	Enclosure of Primary Tanks and Odor Control		200,000
	Phosphorous Removal Facilities		75,000
	Effluent Filters and Aerators		450,000
	Sludge Handling		250,000
	Miscellaneous		55,000
			1,250,000*

*Exclusive of engineering and contingencies

TABLE I-4

SOUTH BUFFALO STP - UNIT UPGRADING COSTS

Date of Upgrading	Unit Upgrading Cost Cost per lb/day Increased BOD_5 Removal	
	$ Cost at Time of Upgrading	1972 $ Cost
1957	343	833
1964-6	31	57
1970	14	18
1972 proposal		145



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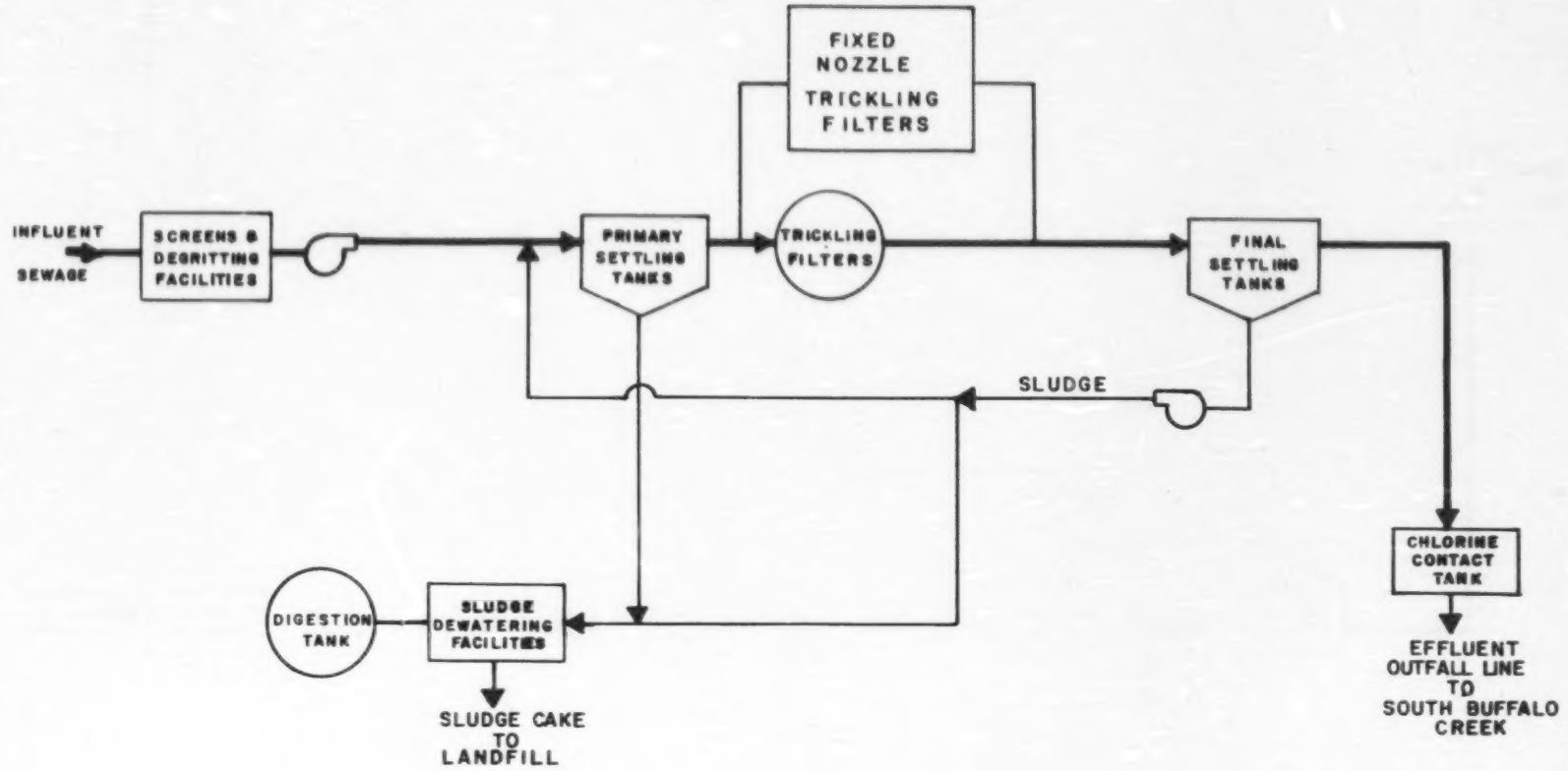
FLOW DIAGRAM NO. 1
AT SOUTH BUFFALO:
ORIGINAL PLANT

FIGURE I-1



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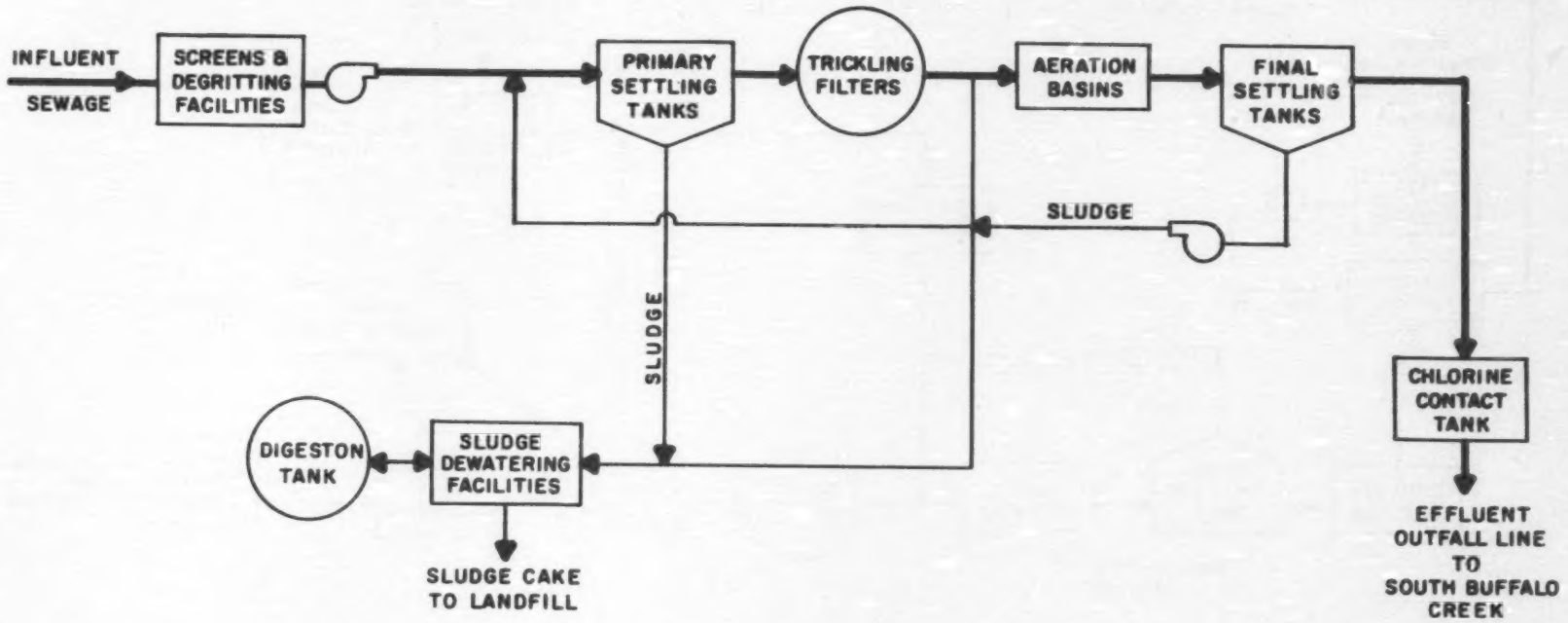
FLOW DIAGRAM NO. 2
AT SOUTH BUFFALO:
FOLLOWING FIRST UPGRADE

FIGURE I - 2



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FLOW DIAGRAM NO. 3
AT SOUTH BUFFALO:
FOLLOWING SECOND UPGRADE

FIGURE I-3

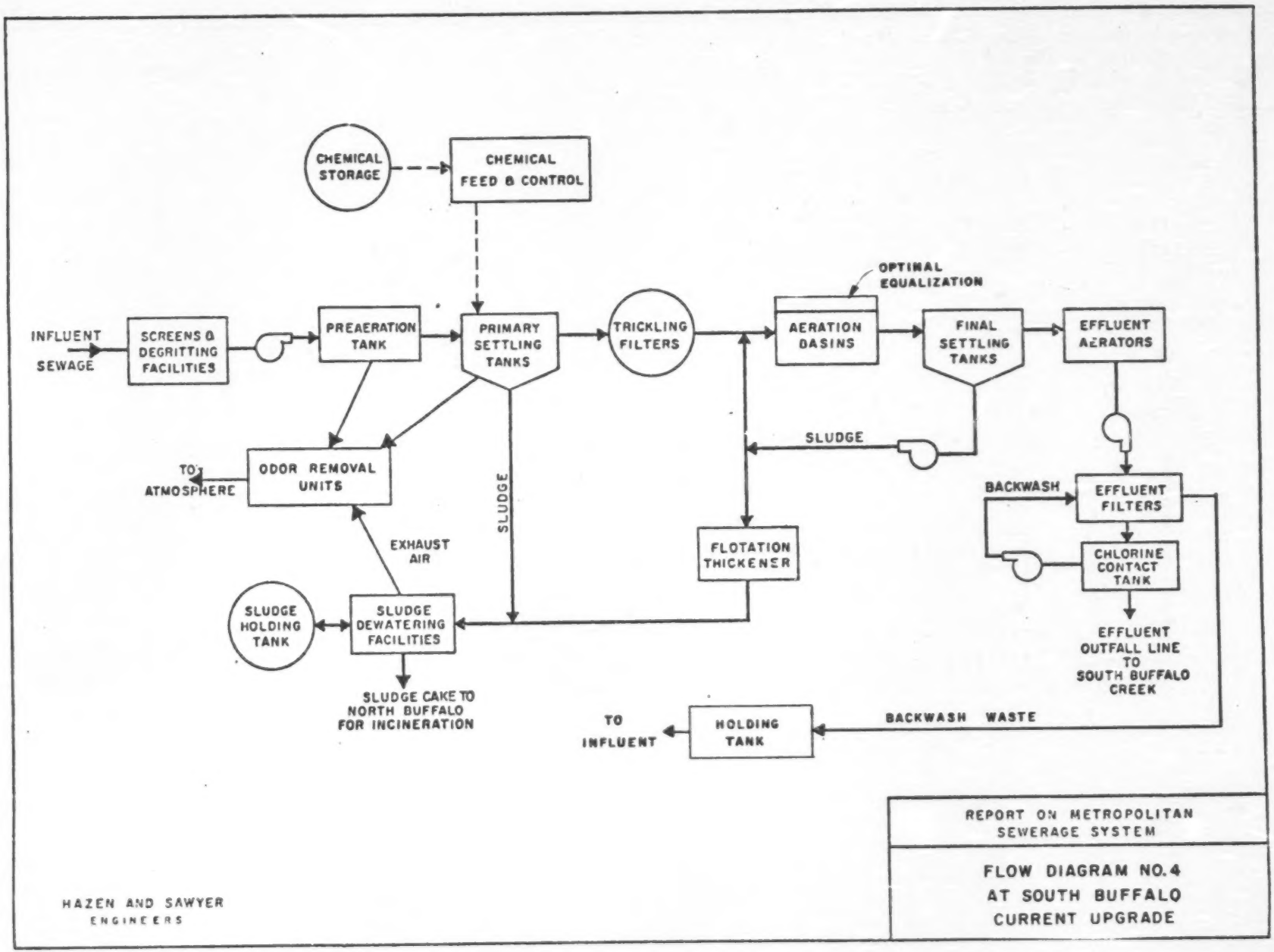

CHEMICAL STORAGE
CHEMICAL FEED & CONTROL
OPTIMAL EQUALIZATION
INFLUENT SEWAGE
SCREENS & DEGRITTING FACILITIES
PREAERATION TANK
PRIMARY SETTLING TANKS
TRICKLING FILTERS
AERATION BASINS
FINAL SETTLING TANKS
EFFLUENT AERATORS
SLUDGE
TO ATMOSPHERE
ODOR REMOVAL UNITS
SLUDGE
BACKWASH
EFFLUENT FILTERS
CHLORINE CONTACT TANK
FLOTATION THICKENER
EXHAUST AIR
SLUDGE HOLDING TANK
SLUDGE DEWATERING FACILITIES
EFFLUENT OUTFALL LINE TO SOUTH BUFFALO CREEK
SLUDGE CAKE TO NORTH BUFFALO FOR INCINERATION
TO INFLUENT
HOLDING TANK
BACKWASH WASTE
HAZEN AND SAWYER ENGINEERS
REPORT ON METROPOLITAN SEWERAGE SYSTEM
FLOW DIAGRAM NO. 4 AT SOUTH BUFFALO CURRENT UPGRADE

FIGURE I-4

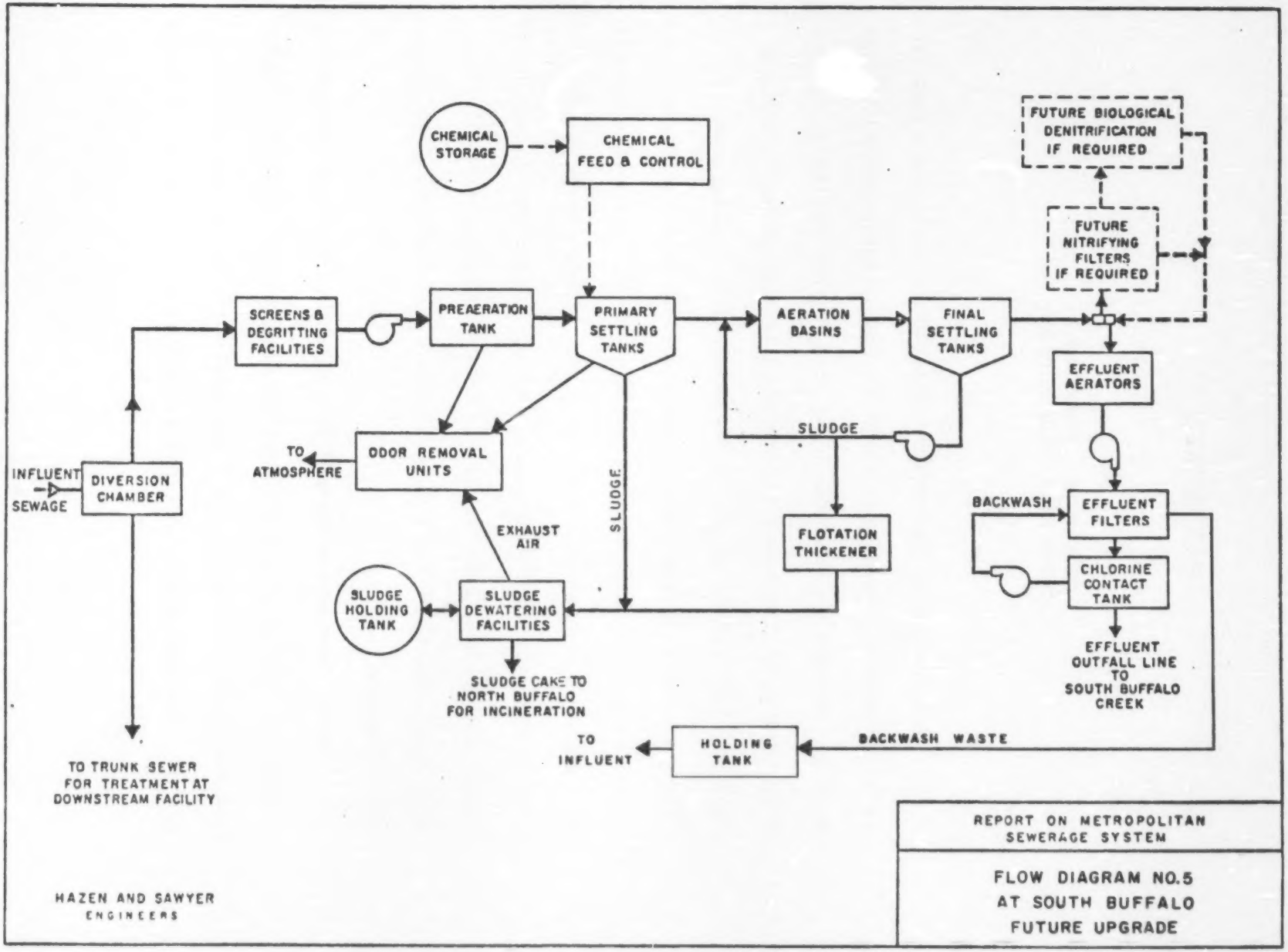

CHEMICAL STORAGE
CHEMICAL FEED & CONTROL
FUTURE BIOLOGICAL DENITRIFICATION IF REQUIRED
FUTURE NITRIFYING FILTERS IF REQUIRED
SCREENS & DEGRITTING FACILITIES
PREAERATION TANK
PRIMARY SETTLING TANKS
AERATION BASINS
FINAL SETTLING TANKS
EFFLUENT AERATORS
INFLUENT SEWAGE
DIVERSION CHAMBER
TO ATMOSPHERE
ODOR REMOVAL UNITS
SLUDGE
BACKWASH
EFFLUENT FILTERS
EXHAUST AIR
SLUDGE
FLOTATION THICKENER
CHLORINE CONTACT TANK
SLUDGE HOLDING TANK
SLUDGE DEWATERING FACILITIES
EFFLUENT OUTFALL LINE TO SOUTH BUFFALO CREEK
SLUDGE CAKE TO NORTH BUFFALO FOR INCINERATION
TO INFLUENT
HOLDING TANK
BACKWASH WASTE
TO TRUNK SEWER FOR TREATMENT AT DOWNSTREAM FACILITY
HAZEN AND SAWYER ENGINEERS
REPORT ON METROPOLITAN SEWERAGE SYSTEM
FLOW DIAGRAM NO.5 AT SOUTH BUFFALO FUTURE UPGRADE

FIGURE I-5

CASE HISTORY II

LIVERMORE, CALIFORNIA
WATER RECLAMATION PLANT

The Livermore Wastewater Treatment Plant was built in 1958 to provide secondary treatment for a domestic wastewater with an average dry weather flow of 2.5 million gallons per day. The plant included preliminary treatment and roughing filters followed by 30-day oxidation ponds. Sludge was digested, then dewatered in lagoons. Effluent was discharged to an intermittently flowing drainage ditch. See Figure II-1 for plant flow diagram, Table II-1 for plant design data. Design effluent limitations were 40 mg/l BOD_5 and 40 mg/l suspended solids. The plant achieved effluents with 45-50 mg/l BOD_5 and 45-50 mg/l suspended solids, with no significant nitrification.

The plant was enlarged in 1967 to provide an increased degree of treatment for an average dry weather flow of 5 mgd. All existing structures were incorporated in the upgraded plant. The new plant includes preliminary treatment, roughing

filters, activated sludge secondary treatment, pre- and post-chlorination. The former oxidation ponds have been converted to the emergency holding ponds. The existing sludge disposal system was expanded. Part of the effluent (about 20 per cent) is used to irrigate a municipal golf course, an airport and adjacent farm land. See Figure II-2 for plant flow diagram, Table II-2 for plant design data. Effluent limitations (Regional Water Quality Control Standards) were 20 mg/l BOD_5, 20 mg/l suspended solids, 1 mg/l grease, 5 MPN/100 ml coliforms (using 5-day median). The plant easily accomplishes this (see Table II-3 for operating data). Complete nitrification must be provided to achieve the low effluent bacterial count (if nitrification is not complete, chlorine demand is inordinate because of chloramine formation, and inadequate free chlorine remains to effect the requisite high levels of disinfection).

The initial plant had a capital cost of $900,000 (1957 dollars) and an annual operating cost of $102,000 (1968 dollars). Capital cost of the expansion was $1,300,000 (1968 dollars). Annual operating cost is $270,000 (1972 dollars). Unit capital costs of upgrading are $520,000 per mgd of increased capacity, or $186 per lb/day of additional BOD_5 removal.

TABLE II-1

CITY OF LIVERMORE, CALIFORNIA
WASTEWATER TREATMENT PLANT
DESIGN DATA, 1958

Design Flow	
Average dry weather, mgd	2.5
Maximum dry weather, mgd	4.5
Peak storm rate, mgd	10.0
Design Loadings	
Population equivalent	28,900
Suspended solids, pounds per day	
Per capita	0.15
Design total	4,500
Biochemical oxygen demand, pounds per day	
Per capita	0.20
Design total	6,500
Preaeration Tanks	
Number[a]	2
Width, feet	19
Length, feet	38
Average water depth, feet	11.7
Detention time, hours (one tank)	0.6
Air supplied per tank, cfm[b]	200
Air supplied, cf per gallon	0.12
Maximum hydraulic capacity, mgd	10
Primary Sedimentation Tank	
Width, feet	19
Length, feet	124
Average water depth, feet	9
Effluent weir length per tank, feet	164
Detention time, hours	1.5
Mean forward velocity, feet per minute	1.4
Overflow rate, gallons per sfpd at avg DW flow	1,050
Max. hydraulic capacity of tank, mgd	10
Assumed removal, per cent	
Suspended solids	60
Biochemical oxygen demand	35

(a) One preaeration tank used to aerate trickling filter effluent prior to secondary sedimentation.

(b) 200 cfm supplied to primary aeration tank and 100 cfm supplied to secondary aeration tank.

Secondary Sedimentation Tank	
Details: as for primary (except removals)	
Trickling Filter	
Inside diameter, feet	110
Average depth of filter media, feet	4.25
Size of filter media, inches	2 to 4
Net area of filter surface, acres	0.218
Volume, acre feet	0.92
Circulation ratio to avg design DWF	1.5 to 3.0
Loading	
Rate per filter, mgd	7.5
Rate per surface acre, mgd	34.5
BOD, pounds per day per acre-foot	4,600
Assumed removal, per cent, filter plus secondary sedimentation	
Suspended solids	60
Biochemical oxygen demand	75
Oxidation Ponds	
Total area, acres	37
Average water depth, feet	6
Detention time, days	30
BOD loading, lbs per acre per day	28

TABLE II-2

CITY OF LIVERMORE, CALIFORNIA
WATER RECLAMATION PLANT
DESIGN DATA, 1968

Flow	
Average dry weather, mgd	5.0
Maximum dry weather, mgd	10.0
Peak storm rate, mgd	18.0
Design Loadings	
Population equivalent	62,500
Suspended solids, lbs per day	
Per capita	0.20
Total	12,500
Biochemical oxygen demand, lbs per day	
Per capita	0.20
Total	12,500
Preaeration Tanks	
Number	2
Width, feet	19
Length, feet	38
Average water depth, feet	11.7
Detention time, hours	0.6
Air supplied per tank, cfm	150
Hydraulic capacity, mgd	10
Primary Sedimentation Tanks	
Number	2
Width, feet	19
Length, feet	124
Average water depth, feet	9
Effluent weir length per tank, feet	164
Detention time, hours	1.5
Mean forward velocity, fpm	1.4
Overflow rate, gal per sq ft per day at ADWF	1,050
Hydraulic capacity, mgd	10
Primary Treatment	
Assumed suspended solids reduction, per cent	60
Suspended solids reduction, lbs per day	7,500
Assumed BOD reduction, per cent	35
BOD reduction, lbs per day	4,400

Trickling Filters	
Number	2
Inside diameter, feet	110
Average depth, filter media, feet	4.25
Size of filter media, inches	2 to 4
Net area of filter surface, acres	0.218
Volume, acre-feet per filter	0.92
Circulation ratio to ADWF	1.5 to 3.0
Loading	
Rate per filter, mgd	7.5
Rate per surface acre, mgd	34.5
BOD, lbs per day per acre-foot	4,400
Assumed BOD removal, per cent	50
BOD reduction, lbs per day	4,000
Activated Sludge Aeration Tanks	
Number	2
Average water depth, feet	15
Width, feet	30
Length, feet	160
Detention time, hours, based on raw sewage flow	5.2
Air supplied, cu ft per lbs BOD removed, cfm	1200
Volume per tank, cu ft	72,000
Volumetric loading, lbs BOD per 1000 cu ft	28
Return sludge, per cent	10 -100
Secondary Sedimentation Tanks	
Number	1
Diameter, feet	90
Side water depth, feet	12
Detention time, hours, based on ADWF	2.75
Overflow rate, gal per sq ft per day at ADWF	787
Activated Sludge Treatment	
Assumed suspended solids reduction, per cent	88
Suspended solids reduction, lbs per day	4,400
Assumed BOD reduction, per cent	86
BOD reduction, lbs per day	3,500

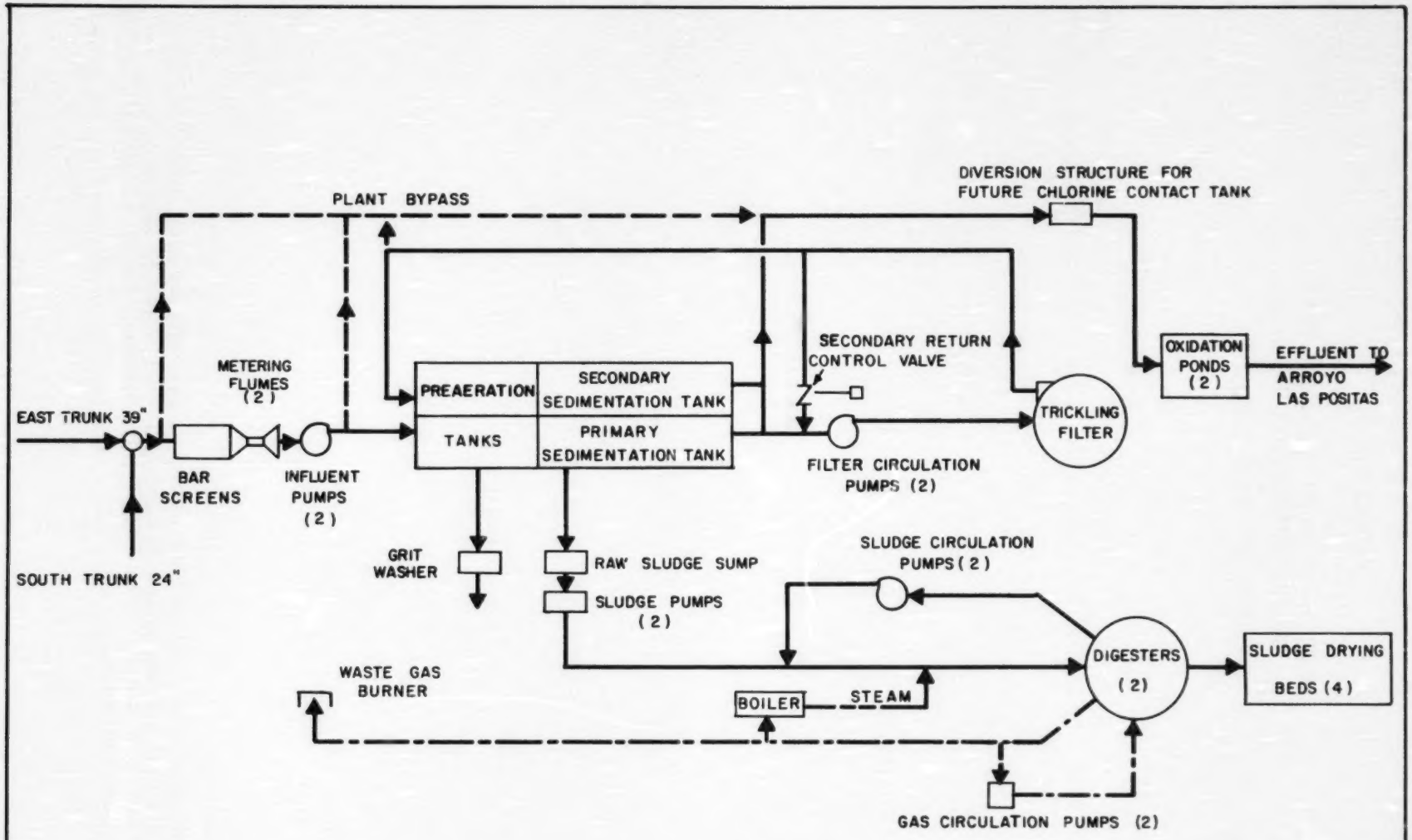
Overall Plant Performance	
Assumed suspended solids reduction, per cent	96
Assumed BOD reduction, per cent	96
Effluent suspended solids, mg/l	15
Effluent BOD, mg/l	15
Chlorine Contact Tank	
Volume, cu ft	28,000
Detention time at ADWF, hours	1.0

TABLE II-3

CITY OF LIVERMORE, CALIFORNIA
WATER RECLAMATION PLANT
OPERATING DATA, 1971[1]

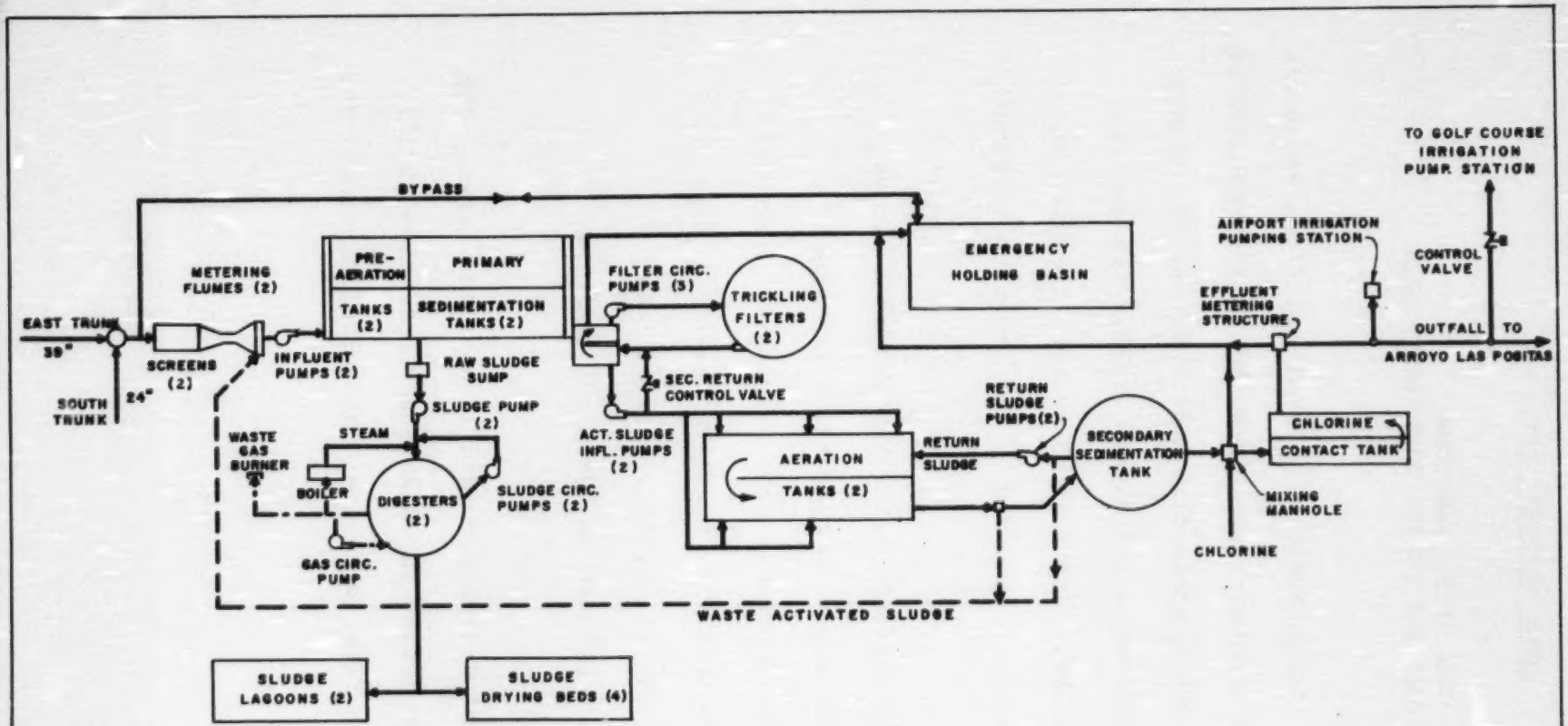
Item	Raw Influent	Primary Effluent	Trickling Filter Effluent	Secondary Effluent	Plant Effluent
BOD (mg/l)					
Mean	213	127	93	9.9	7.3
High	244	156	129	16.9	12.4
Low	167	106	46	5.2	3.1
Suspended Solids (mg/l)					
Mean	229	79	110	22	13
High	261	98	137	31	19
Low	187	60	74	15	8
Ammonia 'N' (mg/l)					
Mean		40.7	32.6	1.68	0.14
High		51.1	44.4	6.73	1.3
Low		33.8	22.5	0.46	<0.1
Nitrate 'N' (mg/l)					
Mean				18.5	21.5
High				20.2	30
Low				16.6	14
Grease (mg/l)					
Mean					0.23
High					0.46
Low					0.15
Coliforms (MPN/100 ml)					
Mean					2.5
High					7.0[2]
Low					0.4

(1) Monthly average. If value not given, item not tested for.
(2) When coliform count exceeded 5.0 MPN/100 ml, flow diverted to emergency holding pond. Discharge not resumed until count of less than 5.0 achieved.



CITY OF LIVERMORE, CALIFORNIA
WASTEWATER TREATMENT PLANT
FLOW DIAGRAM, 1958

FIGURE II-1



CITY OF LIVERMORE, CALIFORNIA
WATER RECLAMATION PLANT
FLOW DIAGRAM, 1968

FIGURE II - 2

CASE HISTORY III

NEW YORK, NEW YORK
WARDS ISLAND WASTEWATER TREATMENT PLANT

The Wards Island Wastewater Treatment Plant was built in 1937 to provide secondary treatment for an average flow of 180 mgd. By 1947 the average flow was in excess of 226 mgd; solids removal performance had become erratic and air consumption was excessive. These problems were remedied and the capacity increased to 240 mgd by two inexpensive plant modifications:

(a) The conventional activated sludge aerators were converted to operate on the step aeration process which was conceived by Gould for New York City treatment plants. This was the first full plant installation.

(b) The flow pattern in the final tanks was changed.

The original design provided 16 aeration tanks, with sufficient aeration volume for 5.25 hours detention at 125 per cent of average flow, with one tank out of service. The

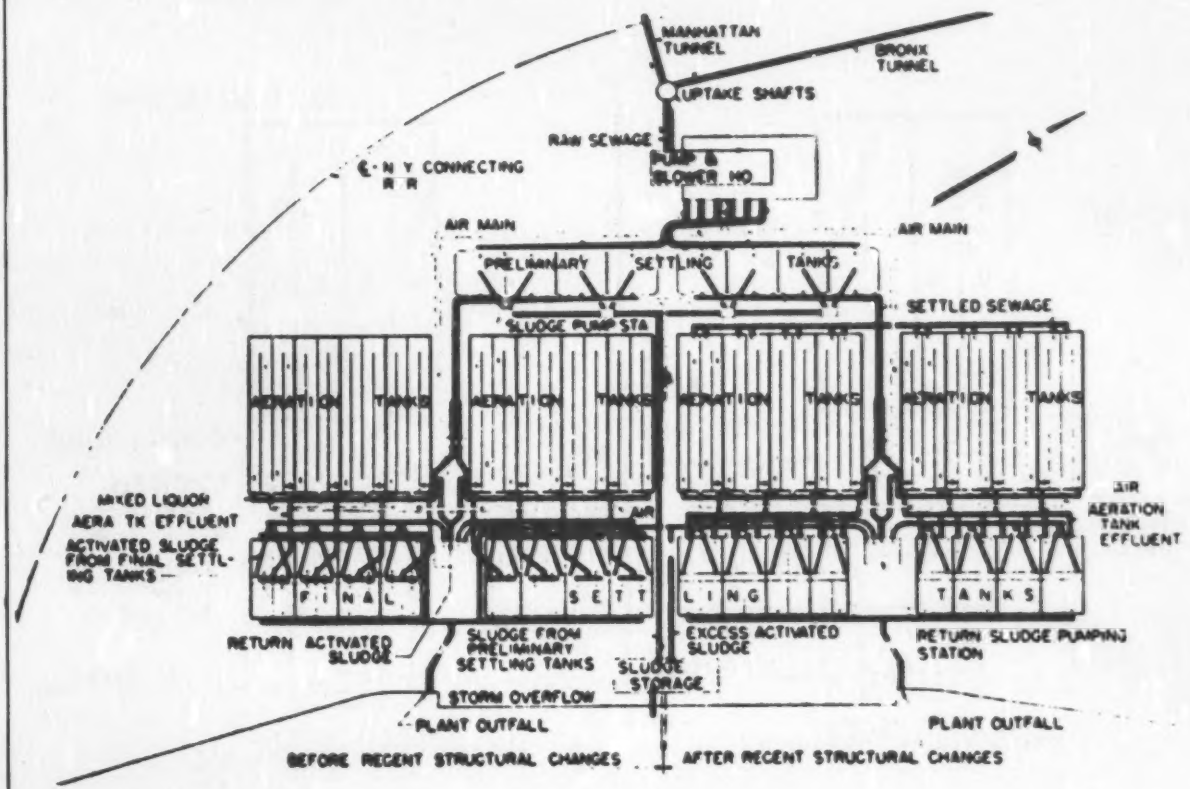
modification of the tank influent systems to provide step aeration (See Figures III-1 and III-2) cut the requisite aeration detention time to 2.5 hours, with no deleterious effect on performance and greatly enhanced operational flexibility. Using the step aeration process, 12 of the original aerators had sufficient capacity for over 240 mgd. A study of plant capacity carried out in 1965 concluded that if the existing preliminary tanks were replaced and the four surplus aerators converted into final settling tanks, the plant could be rated for at least 250 mgd.

There are 32 final tanks at Wards Island each 179 feet long by 425 feet wide; original design overflow rate was 1000 gallons per day at 125 per cent of average flow, with two tanks out of service. Prior to modification, aeration tank effluent was fed from a center conduit and the final tank effluent passed over weirs at both ends. Sludge was withdrawn from a cross collector hopper at the center of the tank. Tank flow was strongly affected by density currents, which carried solids away from the sludge drawoff at a rate approaching the solids handling capacity of the sludge mechanism. This resulted in the formation of sludge blankets

in excess of two feet, and the subsequent long sludge detention under anaerobic conditions increased oxygen demand in the aeration tank and adversely affected the settling qualities of the sludge,with resulting high solids concentrations in the final effluent. Alterations to the tank (see Figure III-3) consisted of abandoning center feed and providing greatly improved influent-effluent structures. Flow now passes the entire length of the tank. Under the modified flow pattern density currents aid the mechanism in moving sludge to the drawoff more rapidly. Solids carryover has been eliminated, and the sludge blanket is consolidated to a depth of only a few inches above the depth of the collecting mechanism flights.

The initial capital cost of the Ward's Island plant was $13,000,000 (1937 dollars). The alterations to the final tanks cost $260,000 (1949 dollars); aeration tank modifications cost $134,000 (1949 dollars). The cost of converting the surplus aerators to final tanks is estimated at $ 1,400,000 (1965 dollars); the unit capital cost of upgrading the secondary treatment capacity of the plant to 250 mgd would be $ 30,000 (1965 dollars) per mgd of increased capacity.

Figure III-1



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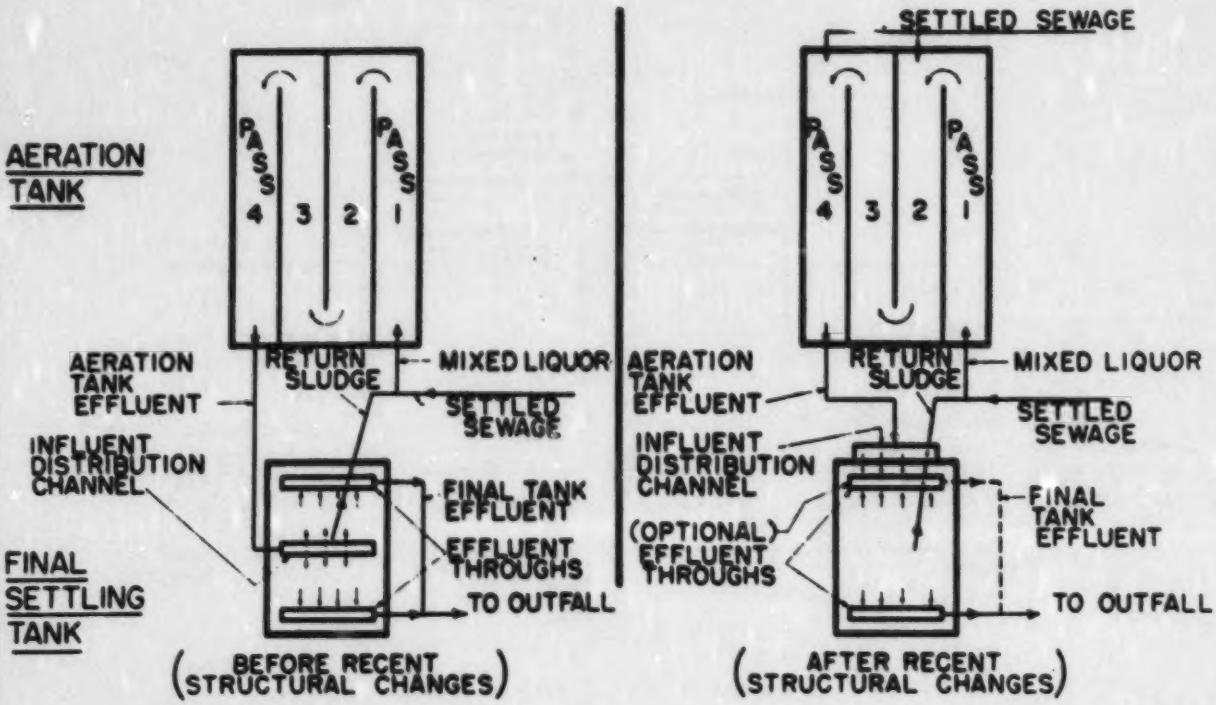
NEW YORK, NEW YORK
WARDS ISLAND WASTEWATER
TREATMENT PLANT
PLANT MODIFICATIONS

Figure III-2



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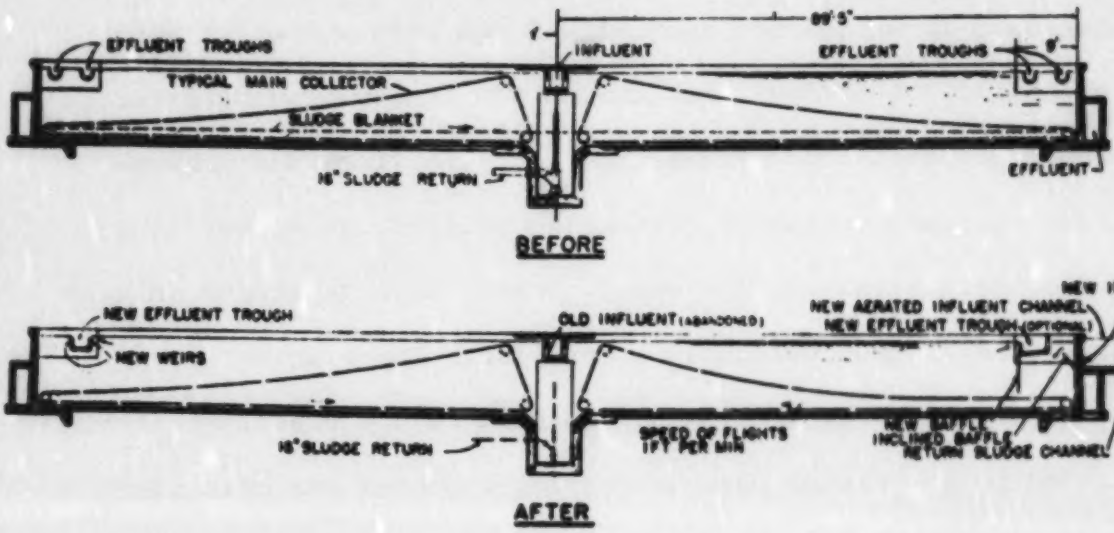
NEW YORK, NEW YORK
WARDS ISLAND WASTEWATER
TREATMENT PLANT
AERATION TANK MODIFICATIONS

Figure III-3



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NEW YORK, NEW YORK
WARDS ISLAND WASTEWATER
TREATMENT PLANT
FINAL TANK MODIFICATIONS

CASE HISTORY IV

UPGRADING EXISTING TRICKLING FILTER PLANT BY ADDITION OF ACTIVATED SLUDGE AHEAD OF FILTERS

This treatment plant, constructed in 1964 at a cost of $1,500,000 (1964 dollars), was designed to give secondary treatment to an average flow of 4.8 mgd with a nominal BOD_5 load of 12,000 lb/day, including 8,500 lb/day of BOD_5 expected from a proposed food processing plant. It included conventional primary treatment, trickling filters, final settling and chlorination. Sludge was digested, then dewatered on sand drying beds. See Figure IV-1 for plant flow diagram.

By 1965 the total waste loading to the treatment plant proved to be greater than originally expected and operating problems developed. Odors developed at preliminary tanks and the trickling filters. The final settling tank sludge was dilute, and would not settle well. Fine solids carried over from the final tanks to the effluent. The dilute final tank sludge was pumped to the plant influent for resettling in the preliminary tanks, whose effluent then also contained fine solids. The mixed sludge from the preliminary tanks

was also dilute and the excess water caused reduced detention time and poor stabilization in the plant anaerobic digester.

Initial modifications made at the treatment plant included: dilution of raw wastewater with stream flow to increase its oxygen content and reduce its strength; covering of the preliminary tanks and trickling filter with ozonation of the exhaust gases for odor control; pumping of final tank sludge to a separate thickening tank, before repumping to the digesters.

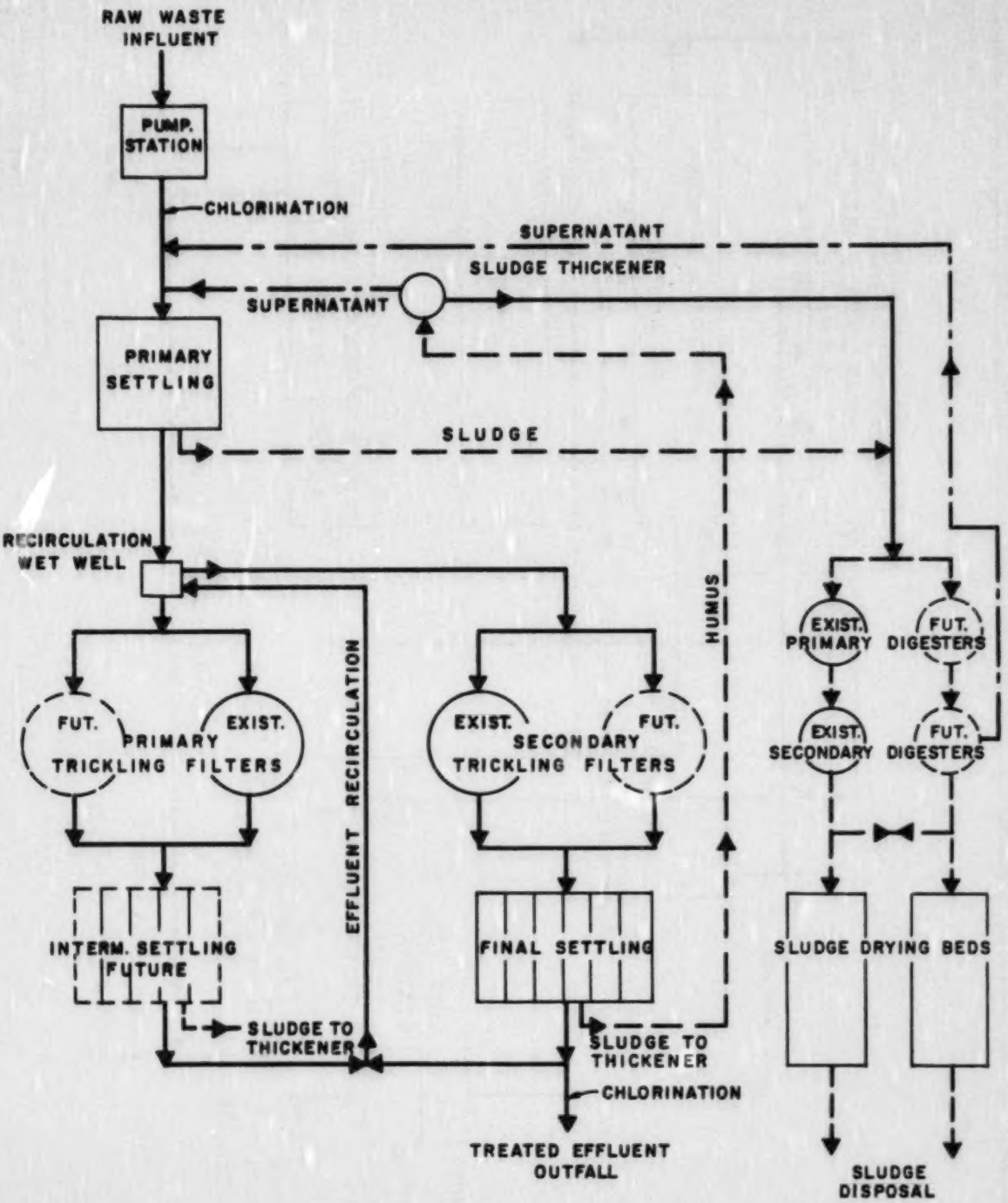
A survey of the processing plant waste in early 1967 showed an average work day flow of 1.25 mgd containing 13,500 lbs. of BOD_5, consisting chiefly of dissolved sugars and similar carbohydrates. Laboratory and pilot plant tests indicated that the waste was amenable to treatment by activated sludge, if nutrients were added. It was decided that the most economical form of additional treatment capacity would utilize the activated sludge process with lagoon type aeration basins.

Because of lack of space at the treatment plant for such units they were constructed upstream near the processing plant, with the effluent discharged to the sewer system.

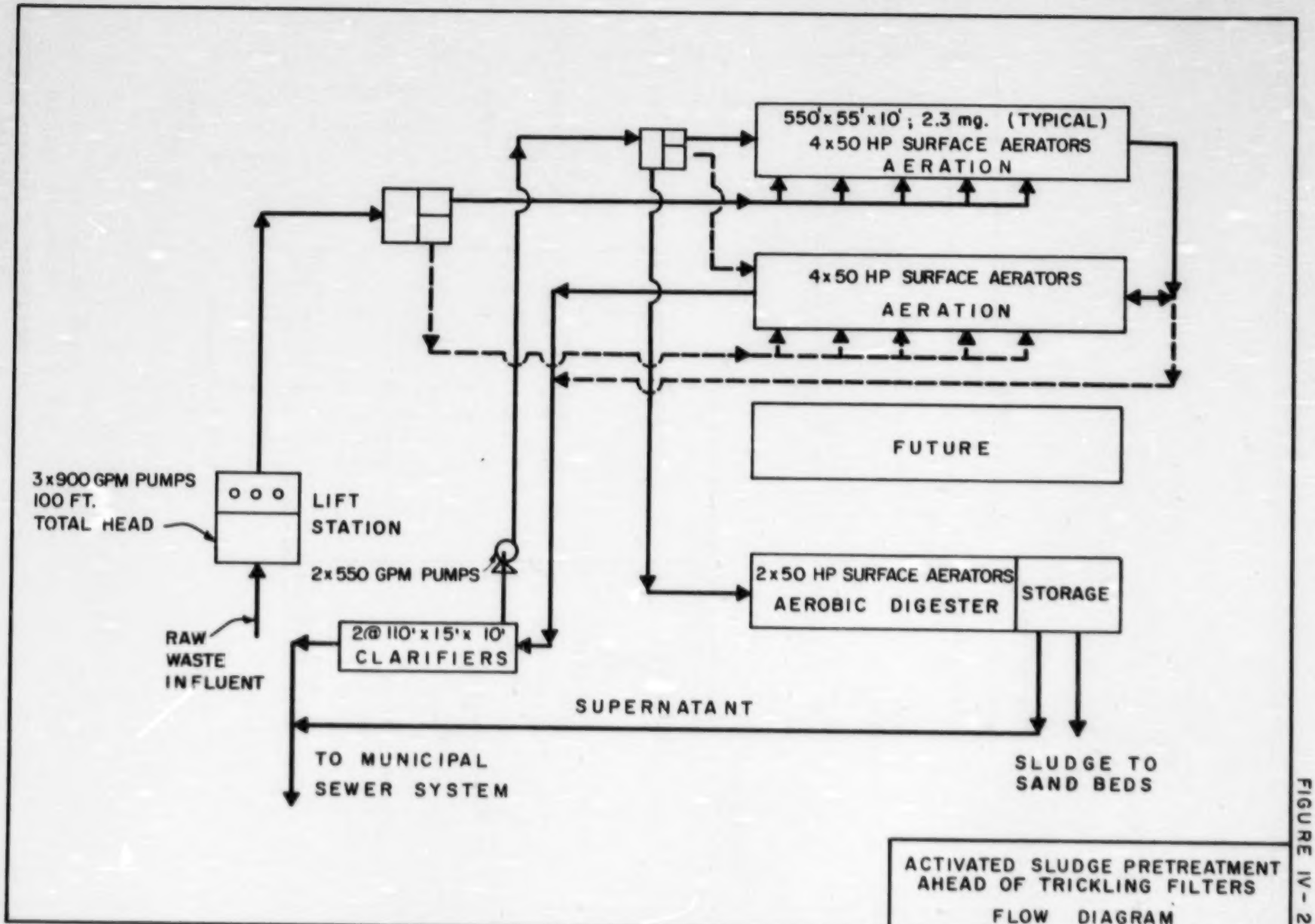
The pretreatment plant was designed for a loading of 1.50 mgd with a BOD_5 of 15,000 lbs/day, and to achieve a BOD_5 reduction of 75 per cent. See Figure IV-2 for plant flow diagram. It went into operation in late 1968 and consistently provides good BOD_5 removal (80-90 per cent), by maintaining a loading of 0.3 to 0.4 lb BOD_5 per lb MLSS at 15 to 22 °C. The reduced loading at the main treatment plant has permitted the plant to operate with a minimum of problems. The sludge from the activated sludge units is filamentous with a generally high sludge volume index and poor settling characteristics. It is stabilized on site in an aerated lagoon and then dried on sand beds. This has greatly relieved the overloaded sludge handling facilities at the main plant.

The additional treatment facility had a capital cost of $ 1,000,000 (1968 dollars); annual operating cost is $ 100,000 (1972 dollars). Unit capital cost of upgrading was $ 83.5 per lb BOD_5 removed (based on 80 per cent BOD_5 reduction).

FIGURE IV-1



TRICKLING FILTER
WASTEWATER TREATMENT PLANT
FLOW DIAGRAM



ACTIVATED SLUDGE PRETREATMENT
AHEAD OF TRICKLING FILTERS
FLOW DIAGRAM

FIGURE IV-2

CASE HISTORIES
REPRINTED FROM EPA TECHNOLOGY TRANSFER DESIGN MANUAL ON UPGRADING EXISTING WASTEWATER TREATMENT PLANTS

13.4 Case History No. 3

Use of Roughing Filter to Upgrade an Existing Low-Rate Trickling Filter Plant (3) (4) (5)

Case History No. 3 involves the upgrading of an existing low rate trickling filter plant in Huber Heights, Ohio. The original plant was designed in August, 1956 for a flow of 0.7 mgd, with 85 percent BOD and suspended solids removals. The community developed so rapidly that by 1970, the average flow had increased to 2.3 mgd. The flow diagrams for the original and upgraded plant are shown in Figure 13-3.

Operational and performance data for the overloaded plant for 1962, when the plant was receiving 1.15 mgd, are compared with corresponding data after the plant was upgraded to 2.3 mgd in Table 13-7.

The comminuter and primary clarifiers were replaced with three Hydrasieve units of 1-mgd capacity each. These units are stationary screens capable of removing 20 to 35 percent of the BOD and suspended solids. A Hydrasieve unit is illustrated in Figure 13-4, along with a schematic flow diagram through the unit. These screens generally require no power and little maintenance.

The plastic media roughing filter used in the upgrading has an application rate of approximately 2.5 gpm/sq.ft. Present BOD removal is about 25 to 35 percent through the roughing unit. Because of the increased hydraulic loading, it was necessary to expand the secondary clarification and chlorine contact tank capacities. The abandoned primary clarifiers were converted into sludge thickeners. This step, in addition to conversion of the anaerobic digester to a high-rate unit using gas recirculation for mixing, enabled the sludge handling system to process the increased quantity of sludge produced.

This case history points out the fact that an existing plant may be gradually upgraded to handle a three-fold increase in flow with the use of innovative techniques and newly applied process equipment. The capital costs for upgrading the capacity of the plant were estimated at approximately $300,000 (ENR 1500), or $187 per 1,000 gallons/day of upgraded flow capacity.

FIGURE 13-3

CASE HISTORY NO.3

COMPARISON OF ORIGINAL AND UPGRADED FLOW DIAGRAMS

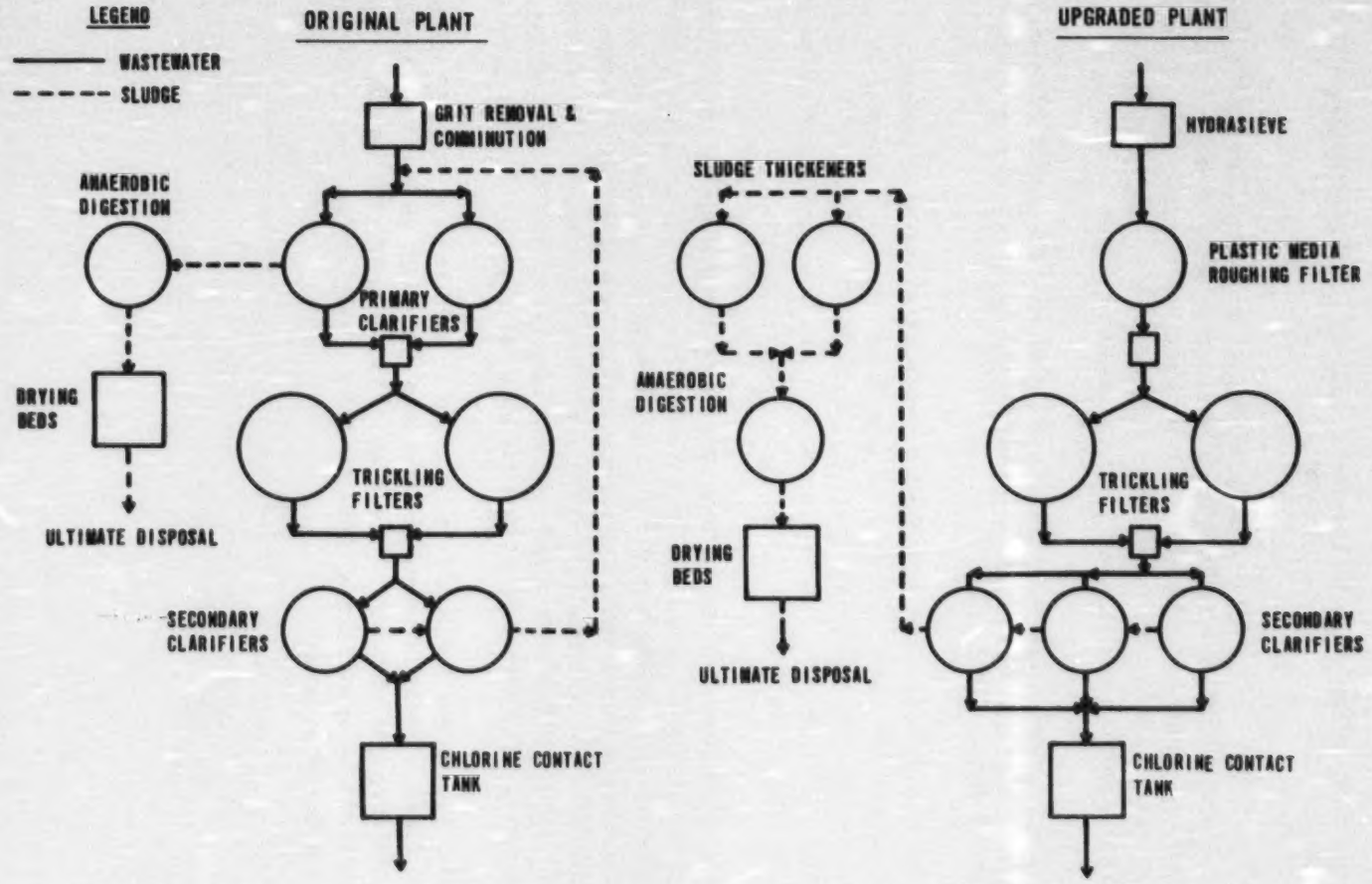


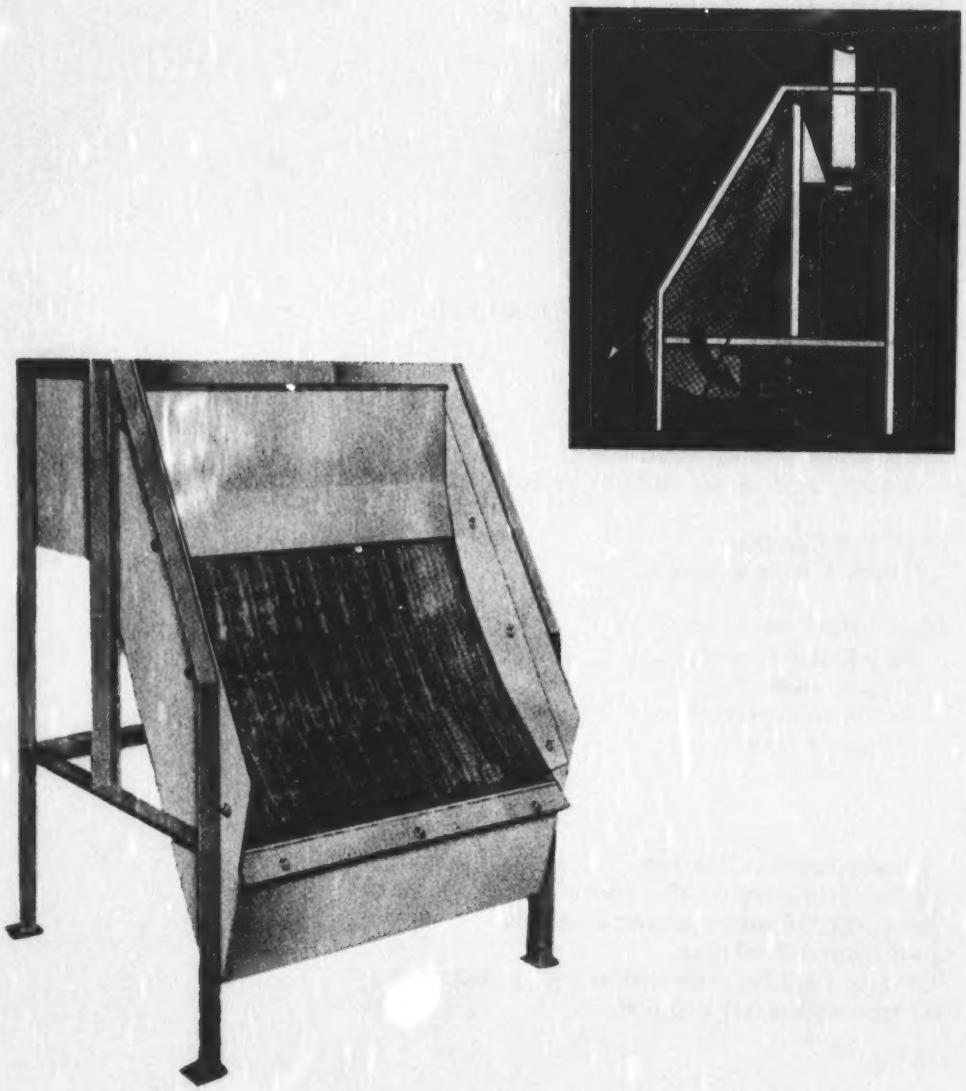
Table 13-7

Case History No. 3 - Plant Operating and Performance Data

Operation	Before Upgrading (1962)	After Upgrading
Average Flow Rate, mgd	1.15	2.3
Primary Clarifier		
Overflow Rate[1], gpd/sq.ft.	1,170	– [5]
Hydrasieve Slot Size, inches	–	0.06
BOD Removal, percent	35[2]	25[3]
SS Removal, percent	– [6]	22[3]
Plastic Media Roughing Filter		
Hydraulic Loading[4], gpm/sq.ft.	–	2.5
Organic Loading[4], lbs. BOD/day/1,000 cu.ft.	–	520
Recirculation Ratio	–	≈2.0
BOD Removal, percent	–	30
Trickling Filter (Stone Media)		
Hydraulic Loading, mgad	6.0	12.0
Organic Loading, lbs. BOD/day/1,000 cu.ft.	56.2	87.0
Secondary Clarifiers		
Overflow Rate, gpd/sq.ft.	1,170	750
Overall Plant Performance		
BOD Removal, percent	83	85
Effluent BOD, mg/l	41	37
SS Removal, percent	– [6]	84
Effluent SS, mg/l	– [6]	40

[1]Based on average flow rate.
[2]Based on primary clarifier performance.
[3]Based on hydrasieve performance only.
[4]Including recirculation.
[5]Primary clarifiers converted to gravity thickeners.
[6]Operating data not available.

FIGURE 13-4
HYDRASIEVE SCREENING UNIT*



*COURTESY OF THE BAUER BROS. CO. - SPRINGFIELD, OHIO

13.5 Case History No. 4

Upgrading an Existing High-Rate Trickling Filter by Conversion to a Super-Rate Filter System (6)

The North Treatment Plant at Sedalia, Missouri, is a high-rate trickling filter plant, designed for 1.25 mgd, and was removing 85 percent of the BOD in 1963. However, the Water Pollution Board set a final effluent BOD of 20 mg/l, which Sedalia could not meet with the existing facilities. The 1963 plant flow diagram is illustrated in Figure 13-5.

The plant was upgraded to treat an average design flow of 2.5 mgd. The existing stone-media filter was renovated to operate with plastic media. In addition, a second plastic-media filter was constructed. The two plastic-media filters are operated in parallel with a total recirculation ratio of 1.55. One additional primary clarifier and one additional secondary clarifier were installed.

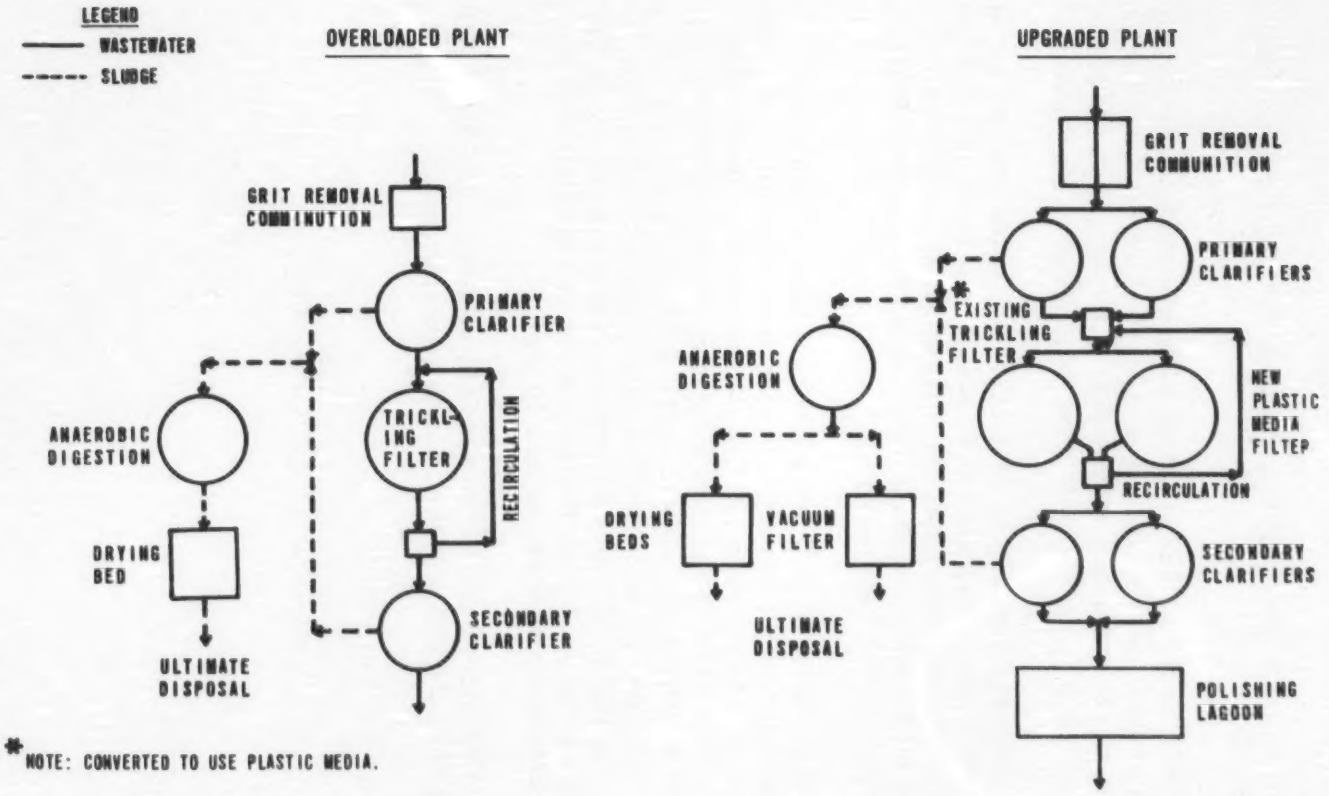
To remove additional BOD and suspended solids, a shallow aerobic polishing lagoon was constructed after the secondary clarifiers, and a vacuum filter was added to reduce the volume of digested sludge. A flow diagram of the upgraded plant is also shown in Figure 13-5. Table 13-8 contains a summary of operating data for the 1963 overloaded period; in addition, the upgraded design criteria are listed, along with actual operational data for the post-upgrading period. It should be noted that the effluent BOD was improved from 115 mg/l to 11 mg/l after upgrading, which is below the 20 mg/l requirement. It should also be pointed out that Missouri has no suspended solids removal requirements for plants with a flow of less than 10 mgd.

The capital costs of upgrading the plant were estimated at $2,600,000 (ENR Index 1,500), or $2,080 per 1,000 gpd of incremental upgraded capacity.

FIGURE 13-5

CASE HISTORY NO. 4

COMPARISON OF ORIGINAL AND UPGRADED FLOW DIAGRAMS



*NOTE: CONVERTED TO USE PLASTIC MEDIA.

Table 13-8

Case History No. 4 - Plant Operating and Design Data

Operation	1963 Operating Data	Upgraded Design	1969 Operating Data
Average Daily Flow, gpm	1.25	2.5	1.80
Raw Wastewater BOD, mg/l	768	576	450
Primary Clarifiers			
Overflow Rate[1], gpd/sq.ft.	1,000	1,000	720
BOD Removal, percent	40	40	60
Trickling Filters			
Hydraulic Loading[2], mgad	24.5	31.8	25
Organic Loading[2], lbs. BOD/day/ 1,000 cu.ft.	226	72.6	20
Recirculation Ratio	1.0	1.55	1.55
Final Clarifiers			
Overflow Rate[1], gpd/sq.ft.	755	755	545
Secondary BOD Removal, percent	75	93.2	86.8
Polishing Lagoon (Shallow Aerobic)			
Maximum BOD Loading, lbs. BOD/ acre/day	–	68	30
BOD Removal, percent	–	12	54
Vacuum Filtration Rate, lbs./sq.ft./hour	–	5.0	–[3]
Overall Plant Performance			
BOD Removal, percent	85	96.5	97.7
Effluent BOD, mg/l	115	20	11

[1]Based on average daily flow.
[2]Including Recirculation.
[3]Lack of sufficient operating data.

13.6 Case History No. 5

Upgrading a Contact Stabilization Package Plant to a "Modified" Completely-Mixed Flow Pattern (7)

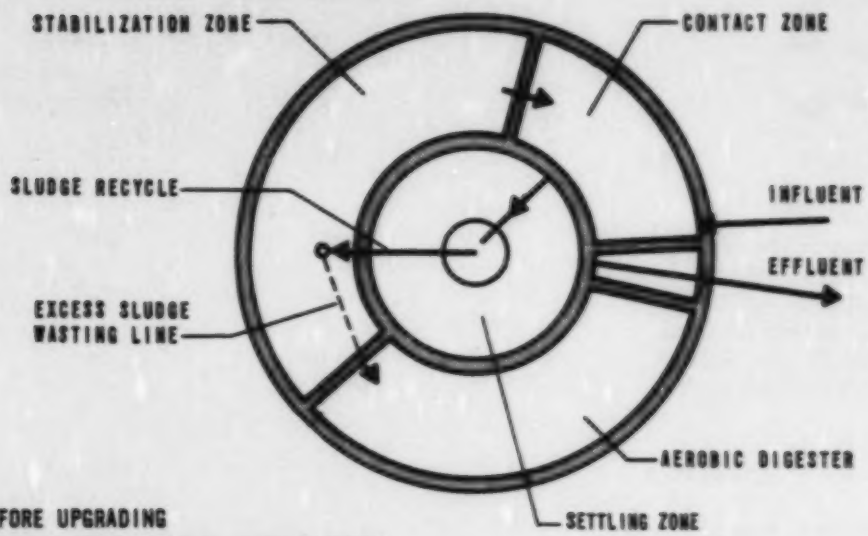
Coralville, Iowa, had a contact stabilization package plant which was providing detention times of 2.6 and 6.5 hours, respectively, in the contact and stabilization zones, based on a forward flow of 867,000 gallons per day, not including sludge recycle. A typical plan view of the contact stabilization package plant is shown in Figure 13-6.

As previously discussed in Chapter 5, a contact-zone detention time of this magnitude may result in improper plant operation because the sludge becomes partially stabilized in this zone and acquires poorer settling characteristics. After investigation, this was found to be the case in Coralville. Operational data from the plant before upgrading are summarized in Table 13-9. The effluent BOD and suspended solids were 26 and 24 mg/l, respectively.

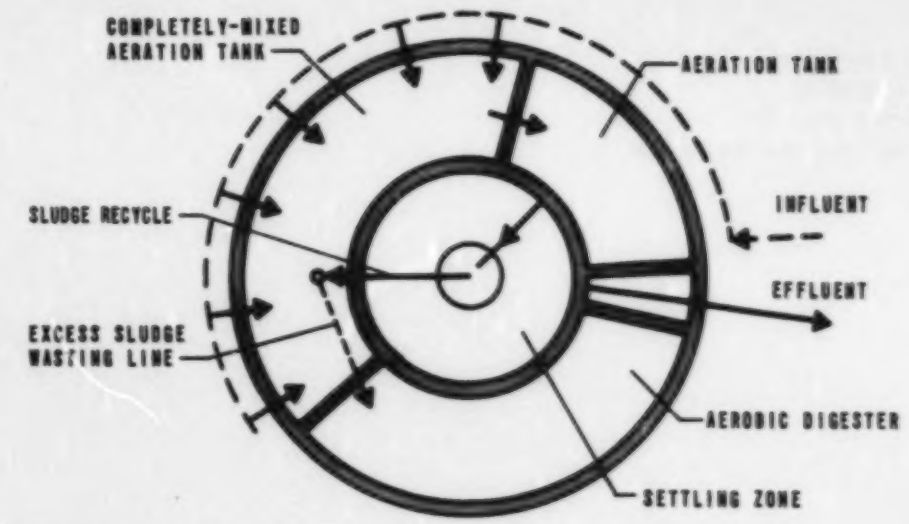
To improve the plant's performance, it was decided to modify the flow pattern as indicated in Figure 13-6. The influent piping was modified so that the raw wastewater was evenly distributed into what originally was the stabilization zone. No wastewater was introduced into the former contact zone. Mixed liquor in the upgraded system proceeded from the former stabilization zone through the former contact zone to the secondary clarifier. The return sludge was introduced into the former stabilization zone at one point only. Therefore, the upgrading resulted in a "modified" completely-mixed flow pattern, with an overall detention time of 9.1 hours for an average flow of 867,000 gpd.

Performance data for the upgraded plant are included in Table 13-9. The effluent BOD and suspended solids concentrations were lowered to 13 and 6 mg/l, respectively, by the upgrading procedure. The costs associated with this modification are primarily due to piping changes. No cost breakdown was available for the modification.

FIGURE 13-6
CASE HISTORY NO. 5
UPGRADING A CONTACT STABILIZATION PACKAGE PLANT
TO A COMPLETELY-MIXED FLOW PATTERN (5)



BEFORE UPGRADING
CONTACT STABILIZATION FLOW PATTERN



AFTER UPGRADING
MODIFIED COMPLETELY-MIXED FLOW PATTERN

Table 13-9

Case History No. 5 - Plant Operating Data

Operation	Contact Stabilization[1]	Modified Completely-Mixed[2]
Average Flow, mgd	0.867	0.867
Influent BOD, mg/l	135	135
Influent SS, mg/l	150	150
Aeration Tank		
Sludge Recycle, percent	60	60
Contact Unit Volumetric Load, lbs. BOD/day/1,000 cu.ft.	78	22[4]
Contact Unit Organic Load, lbs. BOD/day/lb. MLVSS	0.4	-
Contact Unit MLSS, mg/l	3,500	-
Secondary Clarifier		
Overflow Rate[3], gpd/sq.ft.	750	750
Overall Plant Performance		
BOD Removal, percent	81	90
Effluent BOD, mg/l	26	13
SS Removal, percent	84	96
Effluent SS, mg/l	24	6

[1]Before Upgrading
[2]After Upgrading
[3]Based on average flow
[4]Based on total aeration volume.